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SERVICEABILITY PERFORMANCE OF COMPOSITE BEAMS

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Abstract

For composite beams with low degrees of shear connection, additional deflections occur due to slip in the shear connectors, which can be significant for beams with low degrees of shear connection. A design formula is presented for the effective stiffness of composite beams taking account of the stiffness of the shear connectors, which is compared to measured deflections of 6 symmetric beams and an 11m span composite beam of asymmetric profile. It is shown that the comparison is good when using a shear connector stiffness of 70 kN/mm for single shear connectors and 100 kN/mm for pairs of shear connectors per deck rib. Results of push tests on a range of deck profiles confirm these initial elastic stiffnesses. To ensure that the slip at the serviceability limit state does not lead to permanent deformations of the beam, it is proposed that the minimum degree of shear connection should not fall below 30% for un-propped beams and 40% for propped beams of symmetric cross-section.

1. Introduction

The use of composite beams is well established in the UK, especially for office buildings and hospitals where spans of 13 to 20m are required. Long span composite beams are generally designed with a span: depth ratio in the range of 24 to 28 and, for these slender beams, serviceability limits of deflection or vibration sensitivity are the controlling design criteria.

The design of composite beams is presented in BS EN 1994-1-1: Eurocode 4 (BSI, 2004) and in the former BS 5950-3 (BSI,1990) as well as in the AISC 360-10 (AISC, 2010). The rules in these Codes concentrate on the ultimate limit state and on the methods of achieving longitudinal shear connection. Rules for design at the serviceability limit state are more approximate, and the development of improved serviceability rules is the scope of this paper.

In modern composite construction, composite beams are used with composite slabs which span typically 3 to 4m between the beams to form a grillage of secondary and primary beams, as shown in Figure 1. The steel decking used in a composite slab is 50 to 80mm deep and the slab is typically 130 to 160mm in depth.

Figure 1 *Composite beam grillage with profiled decking placed on the beams*

At the ultimate limit state, the design of composite beams is normally based on plastic analysis principles. Where the longitudinal force developed due to the combined resistance of the shear connectors is insufficient to develop the compression resistance of the slab or the tensile resistance of the steel beam, this is known as 'partial shear connection'. The degree of shear connection is defined as a percentage between 100% shear connection for the fully composite beam and zero for the steel beam.

Eurocode 4 presents rules for the minimum degree of shear connection at the ultimate limit state that were developed for propped beams, where all loads are applied to the composite section. The shear connection rules were based on a limiting end slip of 6mm at the plastic resistance of the beam. In Eurocode 4 clause 6.6.1.2, the required minimum degree of shear connection increases linearly with beam span from a minimum of 40% up to 100% at 25m span. In the NCCI guidance (SCI, 2010) and the recent SCI publication 405 (Couchman, 2015), which were produced to complement the use of Eurocode 4 for design in the UK, the minimum degree of shear connection is reduced for un-propped beams and for shear connectors with a higher limiting end slip of 10mm. This was based on evidence from push tests and finite element modelling of composite beams.

However, a further unstated requirement of the minimum degree of shear connection is to ensure that elastic conditions hold at serviceability loads, in order to avoid irreversible deformations under repeated loading. This is expressed as a minimum cut-off in the degree of shear connection that is provided, and in Eurocode 4, the 40% cut-off was based on the analysis of propped beams. The work carried out leading to SCI P405 showed that this cut-off could be reduced for un-propped beams, but should be increased for asymmetric sections (with bottom flange larger than the top flange).

The slip that occurs between the beam and the slab is due to the flexibility of the shear connectors, which adds to deflections under working loads. The former BS 5950-3 gave an approximate formula for the additional deflection as a function of the degree of shear connection, whereas in Eurocode 4, it is not required to take account of additional deflections provided that the minimum degree of shear connection is satisfied and at least

50% shear connection is provided. No guidance is offered in Eurocode 4 as to how to calculate deflections for lower degrees of shear connection.

This paper addresses the effect of partial shear connection on deflections of composite beams and provides a formula for the effective stiffness of composite beams based on the stiffness of the shear connectors. It also defines the minimum degree of shear connection that is required to satisfy elastic conditions at the serviceability limit state. The method is calibrated against the results of both short and long span beam tests and finite element models of composite beams in order to ensure its accuracy for design. This method is 'work in progress' because calibration against more beam tests is required, but it is shown that the results match well to the limited series of beam tests that are investigated.

2. Existing formulae for deflection of composite beams

The deflection of composite beams is calculated from the second moment of area (also known as the inertia) of the composite section, which is generally 3 to 4 times that of the steel beam. This elastic property is calculated for a particular ratio of the elastic moduli of the steel and concrete (modular ratio) on the assumption that the shear connectors are rigid for full shear connection. For a beam that is propped during construction, the long term modular ratio should be used to calculate deflections on removal of the props.

Nethercot and Li (2010) considered the case when yielding of the composite beam may occur at the serviceability limit state for a range of deflection limits and showed the effect of partial yielding along the beam on deflcetions. This was based on high degrees of shear connection and allowed for connection fixity at the ends of the beam. However, in practice, long span composite beams do not yield at serviceability loads and it is the slip in the shear connectors that adds to deflections.

For lower degrees of shear connection, a significant additional deflection occurs due to slip in the shear connectors at the serviceability limit state, and the slip increases as the degree of shear connection reduces. In order to develop simple rules for the effects of partial shear connection on deflections, it is assumed there is a correlation between the slip in the shear connectors at the serviceability limit state and the degree of shear connection at the ultimate limit state.

In BS 5950-3, the additional deflection, w_{add}, due to partial shear connection in un-propped beams is given by a relatively simple equation, as follows:

$$w_{add} = 0.3\,(1-\eta)\,(w_s - w_c) \qquad (1)$$

where

w_c is the deflection of the composite beam for rigid shear connectors (no slip) at the serviceability load. (using the Eurocode symbol, w, for deflection)

w_s is the deflection of the steel beam for the same serviceability loading, and

η is the degree of shear connection of the beam at the ultimate limit state

The coefficient of 0.3 applies to un-propped beams where the self-weight of the slab acts on the steel beam. This coefficient was established from comparisons with relatively short span composite beam tests (Wright and Francis, 1990). For propped beams, the coefficient is taken as 0.5, as the load-slip behaviour of the shear connectors is non-linear and their stiffness reduces with the forces acting on them due to the additional self-weight loads applied on removal of the temporary props.

In the AISC Code, the additional deflection is presented in terms of a modified second moment of area of the composite section, $I_{c,eff}$, given by:

$$I_{c,eff} = I_s + \eta^{0.5} (I_c - I_s) \tag{2}$$

where

I_c is the second moment of area of the composite section for rigid shear connectors.

I_s is the second moment of area of the steel section

Comparisons between the Code methods are given later.

3. Elastic Design of Composite Beams

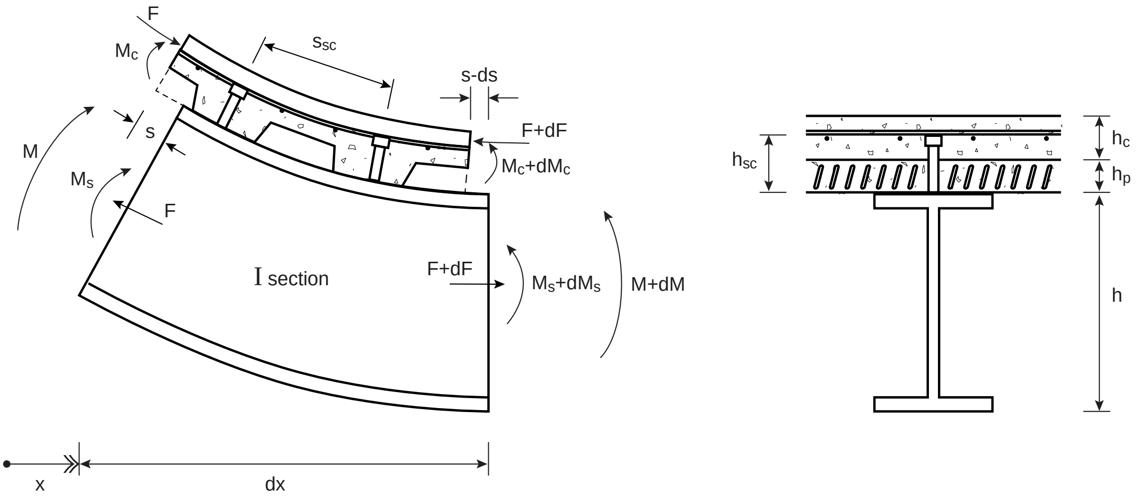
Elastic design is used to check deflections and stresses of composite beams at the serviceability limit state, but may also be used at the ultimate limit state when the cross-section does not meet the Class 1 or 2 criteria in BS EN 1993-1-1: Eurocode 3 (BSI, 2005). Elastic design is also used for composite beams with non–ductile shear connectors that fail to meet the 6mm limiting slip in Eurocode 4.

Slip at the interface between the beam and slab leads to deformations along the beam, as shown in Figure 2. Furthermore, the lower the shear connector stiffness, the higher the deflection due to slip, but the lower the shear forces experienced in the outer shear connectors.

The first theory for the effects of partial shear connection on the design of composite beams was developed by Newmark (Newmark et al, 1951), in which a solution to a differential equation linking slip and deflection for beams with a single point load was presented. The differential equation may also be solved for the general load case using the finite difference method and many papers have covered this approach (e.g. Ranzi and Zona, 2007). Furthermore, the method may be adapted to consider combined shrinkage and slip effects in composite beams (Ranzi et al*.,* 2003)

The following theory shows how the stiffness of the shear connectors may be included in the calculation of the effective stiffness of composite beams and also in the longitudinal shear forces. It is based on the theory presented (Lam et al*,* 2000), which was developed for composite beams supporting precast concrete slabs. In this paper, the additional deflection due to the flexibility of the shear connectors is obtained for a uniformly loaded beam. The theory may be explained by making the assumption that the interface slip follows a simple function along the beam to a maximum slip at the ends of the span. The actual slip distribution along the beam will differ with the loading distribution but comparison with full-scale beam tests shows that simplifications can be made for deflection calculations.

Other simplifying assumptions are that the elastic neutral axis lies in the steel section so that the concrete is un-cracked and also that the effective width of the slab is the same as for the ultimate limit state (= beam span/4). Both of these assumptions are considered to be reasonable approximations for deflection calculations given that the beneficial effects of partial continuity at the connections are not included.



(a) Side view of composite beam showing slip of shear connections

(b) Cross-section through composite beam

Figure 2 *Forces and displacements in a composite beam as affected by slip*

3.1 Elastic Stiffness of Composite Beams – Rigid Shear Connectors

For a composite beam connected to a composite slab, equilibrium of the forces and moments in the cross-section, as shown in Figure 2, is satisfied by:

$$M = M_c + M_s + F(y_s + h_p + 0.5\ h_c) \qquad (3)$$

where M is the externally applied moment

M_c is the moment resisted by the slab

M_s is the moment resisted by the steel beam

F is the longitudinal compression force in the slab, which is balanced by tension in the steel section

y_s is the elastic neutral axis depth of the steel section measured from the top of the steel section

h_c is the depth of the concrete over the deck profile

h_p is the depth of the deck profile

Assuming that there is no separation along the beam and no cracking of the slab, the curvature of the slab and the steel beam is the same at any position, x, and the relative movement between the two determines the rate of change of slip according to:

$$\frac{ds}{dx} = -\frac{M_C}{E_C I_C}\left(h_p + 0.5h_C\right) - \frac{M_S}{E_S I_S}\left(y_s\right) + \left(\frac{F}{E_C A_C} + \frac{F}{E_S A_S}\right) \qquad (4)$$

where $$\frac{M}{E_S I_{eff}} = \frac{M_c}{E_c I_c} = \frac{M_s}{E_S I_S} \qquad (5)$$

and	s	is the slip at any position, x, on the span
	x	is the distance from the support
	A_a	is the cross-sectional area of the steel beam
	A_c	is the cross-sectional area of the concrete slab within the effective width
	E_s	is the elastic modulus of steel
	E_c	is the elastic modulus of concrete
	I_s	is the second moment of area of the steel beam
	I_c	is the second moment of area of the concrete slab
	I_{eff}	is the second moment of area of the composite section taking account of slip

Solving these two equations gives:

$$M = -\frac{ds}{dx}\frac{E_S I_{eff}}{\left(y_S + h_p + 0.5h_c\right)} + \frac{F\ I_{eff}}{\left(y_S + h_p + 0.5h_c\right)}\left(\frac{A_c + nA_s}{A_c A_s}\right) \tag{6}$$

where n is the modular ratio of steel to concrete = E_s/E_c .

For a fully composite beam with rigid shear connectors (with s = 0), the second moment of area is given by:

$$I_{comp} = I_s + I_c/n + (y_s + h_p + 0.5h_c)^2\left(\frac{A_c A_s}{A_c + nA_s}\right) \tag{7}$$

3.2 Elastic Stiffness of Composite Beams - Flexible Shear Connectors

The effective stiffness of a composite beam with flexible shear connectors is established from equation (6). As a good approximation for a uniformly loaded beam, and to simplify the derivation of usable design formulae, the applied moment, M, and compression force, F, are taken as varying according to a sine function, and the slip, s, as varying according to a cosine function with distance x along the beam from a support. The true slip function is more complex, but this simple function is reasonably accurate for beams with uniform loading and is shown to be reasonably accurate for beams with point loads at the one third span points.

The maximum compression force in the slab is determined from the integral of the forces in the shear connectors, which are a function of their elastic stiffness, k, longitudinal spacing, and end slip as follows:

$$\overline{F} = \int_o^x \frac{k}{s_{sc}}\bar{s}\cos\left(\frac{\pi x}{L}\right) = \frac{L}{\pi}\frac{k}{s_{sc}}\bar{s}\sin\frac{\pi x}{L} = \frac{L}{\pi}\frac{k}{s_{sc}} \quad \text{for } x = L/2 \tag{8}$$

and

$$\frac{ds}{dx} = \left(\frac{\pi}{L}\right)\bar{s}\sin\frac{\pi x}{L} \tag{9}$$

where	$\overline{M}$	is the maximum moment at mid-span
	$\overline{F}$	is the maximum force in the concrete slab at mid-span
	$\bar{s}$	is the end slip
	k	is the stiffness of the shear connectors
	s_{sc}	is the longitudinal spacing of the shear connectors
	L	is the beam span

Inserting the above equations into equation (6) gives the following formula linking the end slip and mid-span moment:

$$\bar{s} = \frac{(L/\pi)\left(y_s + 0.5\,h_c + h_p\right)\overline{M}}{EI_{eff}\left[1 + \left(\frac{L}{\pi}\right)^2 . \left(\frac{k/s_{sc}}{E_s}\right) . \left(\frac{A_c + n\,A_s}{A_s\,A_c}\right)\right]} \tag{10}$$

The effective inertia of the composite beam as a function of the shear connector stiffness is:

$$I_{eff} = I_s + \frac{I_c}{n} + \frac{\left(y_s + 0.5\,h_c + h_p\right)^2\left(\frac{A_c}{n}\right)}{\left[1 + \frac{A_c}{nA_s} + \left(\frac{\pi}{L}\right)^2\left(\frac{E_s}{k/s_{sc}}\right)\left(\frac{A_c}{n}\right)\right]} \tag{11}$$

The end slip in equation (10) may be simplified as a function of the inertias of the composite section, I_{comp}, for rigid shear connectors, and of the steel section, I_s, making the assumption that the term I_c/n is small and can be neglected. It is given as follows:

$$\bar{s} = \frac{(L/\pi)\left(y_s + 0.5\,h_c + h_p\right)\overline{M}}{EI_{comp}\left[\frac{I_s}{I_{comp}} + \left(\frac{L}{\pi}\right)^2 . \left(\frac{k/s_{sc}}{E_s}\right) . \left(\frac{A_c + n\,A_s}{A_s\,A_c}\right)\right]} \tag{12}$$

The maximum shear connector force is given by multiplying the end slip by the stiffness, k. The additional mid-span deflection of the composite beam w_{add} due to end slip relative to the deflection of the beam with rigid shear connectors is given by:

$$w_{add} = w_{comp}\left(\frac{1 - \left(\frac{I_s}{I_{comp}}\right)}{\left(\frac{I_s}{I_{comp}}\right) + \left(\frac{L}{\pi}\right)^2\left(\frac{k/s_{sc}}{E_s}\right) . \left(\frac{A_c + n\,A_s}{A_s\,A_c}\right)}\right) \tag{13}$$

where w_{comp} is the deflection of the composite beam based on an inertia of I_{comp}.

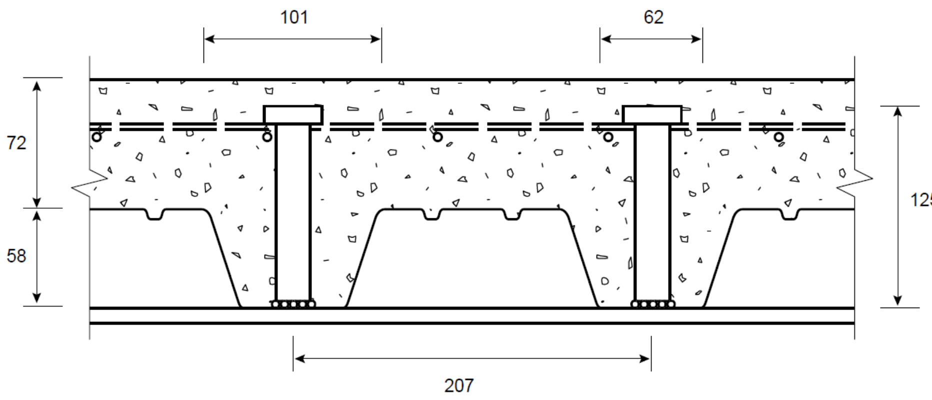
It is not normally required to calculate the additional stresses due to partial shear connection, but in principle, the stresses in the flanges and concrete can be calculated knowing I_{eff}.

4. Study on Shear Connector Stiffness

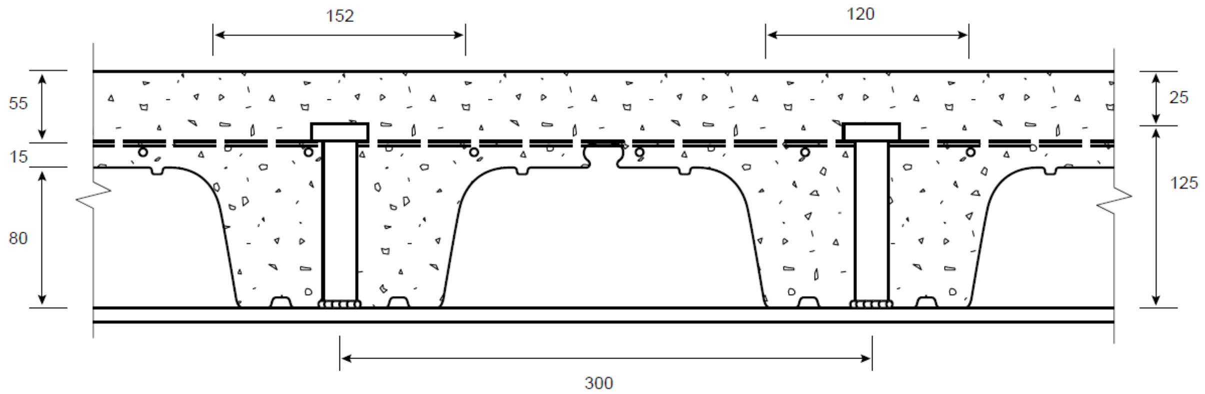
The elastic stiffness of single and pairs of shear connectors was obtained from the results of push-out tests carried out in a recent EU project called DISCCo (see acknowledgement) and also from previous tests. This review does not cover all existing push tests which would have been a major investigation but representative tests from the series were selected for this study. The main emphasis was on 19 mm diameter through deck welded shear connectors placed in the middle of the deck ribs. The two trapezoidal deck profiles tested in the DISCCo project are shown in Figure 3 and covered a range of application. The nominal concrete grade was 30/37 and measured cylinder strengths were approximately 40 N/mm^2.

The 80mm deep deck profile had a wide rib and used 125mm high shear connectors (120mm as-welded height) in which the shear connectors were welded in the middle of the deck rib and were embedded only 40 mm into the concrete topping. The 58mm deep deck profile had a narrow rib and used 125 mm high shear connectors in which the shear connectors were embedded 62 mm into the slab. Further tests were performed on 56mm deep re-entrant decking also using 125 mm high shear connectors.

These results were supplemented by additional tests on 60mm and 80mm deep deck profiles presented in SCI RT 1309 which all used deliberately weaker concrete (C20 cylinder strength) so that the effect of the concrete grade on stiffness could be seen. In a push test, there are generally 4 shear connectors per specimen for single shear connector tests and 8 for pairs of shear connectors per deck rib. In the DISCCO tests, an initial 25 cycles of load up to 40% of the predicted failure load was applied befor testing to failure. The load-slip curves from representative tests are shown in Figures 4 to 8.



(a) 58mm deep decking profile and 125mm high shear connectors



(b) 80mm deep decking profile and 125mm high shear connectors

Figure 3 *Details of the trapezoidal deck profiles used in the push tests and beam tests*

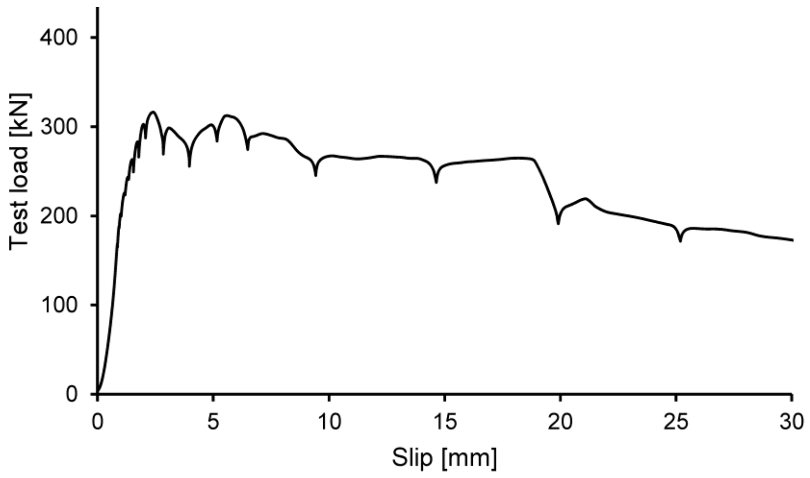


Figure 4 *Push test results with 80mm deep decking using single 125mm high shear connectors*

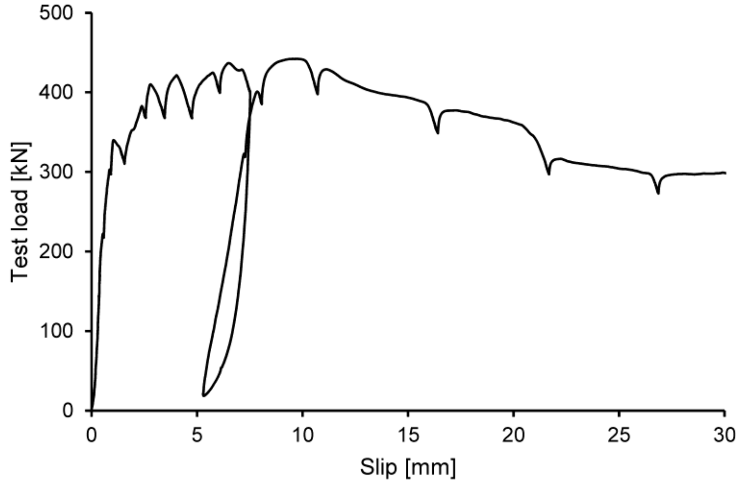


Figure 5 *Push test results with 80mm deep decking with pairs of 125mm high shear connectors*

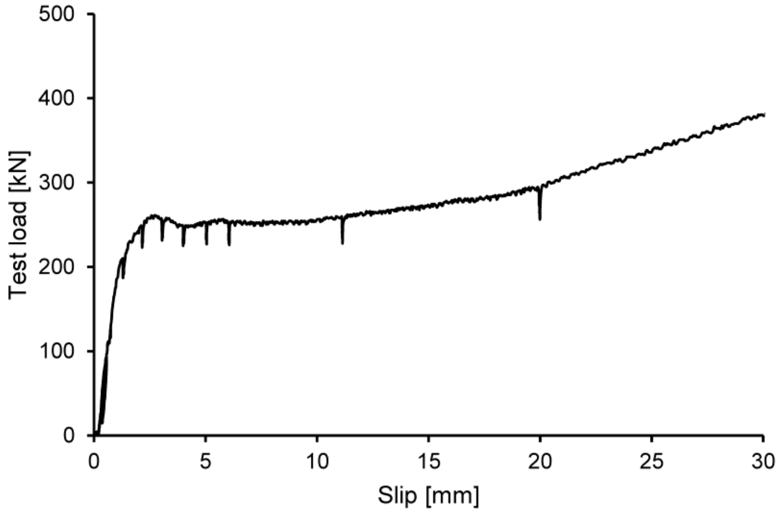


Figure 6 *Push test results with 58mm deep decking using single 125mm high shear connectors*

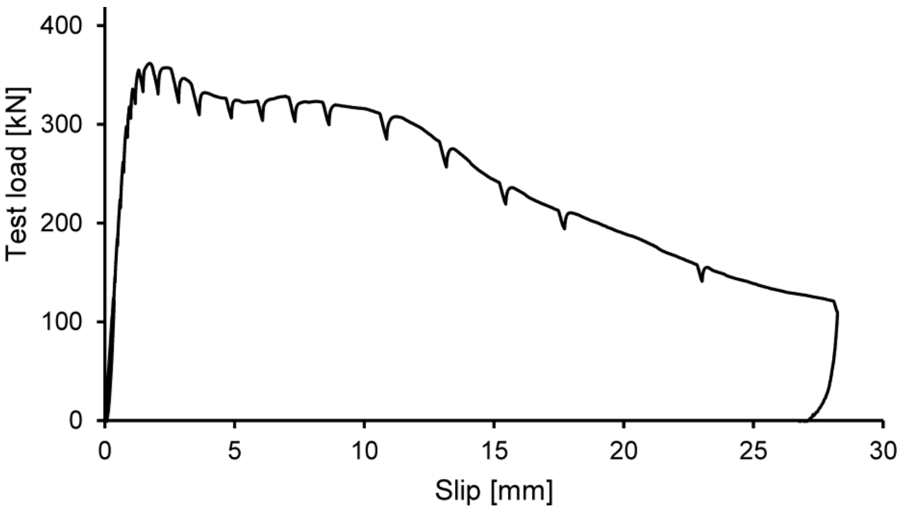


Figure 7 *Push test results with 58mm deep parallel decking and shear connectors at 200mm spacing*

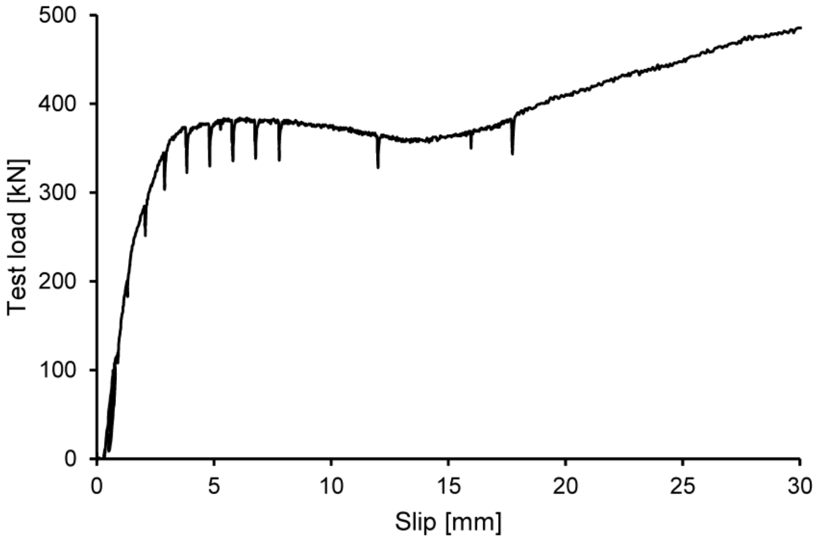


Figure 8 *Push test results with 56mm deep re-entrant decking using single 125mm high shear connectors*

For beams that are un-propped during construction, the relevant stiffness for serviceability calculations is taken as being the slope of the load-slip curve in the range of 0.1 to 0.4 P_k that takes account of the loads acting on the shear connectors due to imposed loading as a proportion of the characteristic shear resistance of the shear connectors, P_k. For beams with higher span: depth ratio, this serviceability load will reduce but 0.4 P_k is a sensible upper bound for interpretation of push tests. The relevant stiffness for propped beams is taken as being the slope of the load-slip curve in the range of 0.1 to 0.7 P_k taking account of the loads acting on the shear connectors due to combined dead and imposed loading.

For pairs of shear connectors, the stiffness is presented per deck rib (i.e. for two shear connectors). Further tests were also carried out on the 58mm deep deck profile orientated parallel to the beam with shear connectors placed at 200mm spacing. It was expected that the shear stiffness for parallel decking would be higher than the equivalent transverse decking case.

The shear connector stiffness obtained from the series of push tests are presented in Table 1 for these two stiffness criteria. In each series of tests, at least 4 tests were performed and the average stiffness is presented. The tests reported in SCI RT 1309 did not include initial cyclic loading to 40% of the predicted failure load whereas the tests in DISCCO included this cyclic loading, which may explain the slight increase in stiffness of the SCI tests under low loading. Because of the nature of serviceability calculations, it is argued that the average push test stiffness is more relevant than a statistically derived value and this is confirmed by comparison with the beam tests.

Decking Configuration (see Figure 3)	**Shear connectors**	**Source of data**	**Number of studs per rib**	**Concrete strength (cylinder)**	**Measured shear connector stiffness per deck rib (rounded to nearest 5 kN/mm)**	
Transverse to beam axis					0.1 to 0.4 P_k	0.1 to 0.7 P_k
80mm deep decking with wide rib	19mm dia. x 125mm high	DISCCo project	1	42 N/mm^2	65	60
			2		100	90
		SCI RT 1309	1	20 N/mm^2	80	45
			2		100	60
60mm deep decking with wide rib	19mm dia. x 100mm high	SCI RT 1309	1	20 N/mm^2	75	65
			2		120	80
58mm deep decking with narrow rib	19mm dia. x 125mm high	DISCCo project	1	40 N/mm^2	70	60
			2		120	100
56mm deep decking with wide re-entrant rib	19mm dia. x 125mm high	DISCCo project	1	40 N/mm^2	100	80
			2		120	100
Decking parallel to beam axis:						
58mm deep decking	19mm dia. x 125mm high	DISCCo project	200mm spacing	49 N/mm^2	110	90

Table 1 *Summary of shear connector stiffness obtained from push tests*

In the early calibration studies for Eurocode 4, the mean stiffness of 19 mm diameter shear connectors in a solid slab using C30/37 concrete was taken as 100 kN/mm (Johnson and Molenstra, 1991). A load-slip function for the shear connectors is given (Aribert, 1997)), as follows:

$$F_s(s) = P_{Rk}\left(1 - e^{-0.7s}\right)^{0.8} \tag{14}$$

where

s is the end slip displacement (mm)

P_{Rk} is the resistance of the shear connector

Using this load-slip function, the equivalent stiffness for a shear connector in a solid slab at a slip of 0.2 mm (typical of a load of 0.4 P_k) is about 80 kN/mm. For a slip of 0.4mm (typical of a load of 0.7P_k), the equivalent stiffness is about 70 kN/mm which is broadly consistent with the test values for profiled decking. Other authors (Qureshi et al, 2011a and 2011b) suggest that for composite slabs, the shear stiffness can be approximated to 40% to 60% of the stiffness in a solid slab depending on the type and depth of decking and the position of the shear connector in the deck rib.

From these tests, representative stiffness for decking orientated transverse to the beam axis is presented in Table 2 and may be taken as 70kN/mm for single shear connectors and 100kN/mm for pairs of shear connectors per deck rib. For higher load levels, consistent with propped construction, the elastic stiffness is lower than these values, but for parallel decking, the stiffness is higher.

Deck profile shape and height (shear connectors welded through the decking in the centre of the rib)	Design shear connector stiffness (kN/mm) for 19mm dia. shear connectors (expressed per deck rib)			
	Single shear connectors		Pairs of shear connectors	
	Un-propped beams	Propped beams	Un-propped beams	Propped beams
Trapezoidal profiles of 60 to 80mm depth	70 kN/mm	60 kN/mm	100 kN/mm	80 kN/mm
Re-entrant profiles of 50 to 60mm depth	100 kN/mm	80 kN/mm	120 kN/mm	100 kN/mm
Decking parallel to beam	100 kN/mm	80 kN/mm	Not applicable	

Table 2 *Proposed design values of shear connector stiffness for deflection calculations of un-propped and propped beams*

5. Comparison of Theory with Tests on Composite Beams

An 11m span asymmetric beam and 5 and 6m span symmetric beams were tested at the Universities of Bradford, Stuttgart and Luxembourg as part of the EU project, DISCCo, to obtain data on the performance of composite beams with less than the minimum degree of shear connection at the ultimate limit state to EN 1994-1-1. The tests also gave good data on the serviceability performance of the beams, which is the focus of this paper and so the test results are presented here in summary.

5.1 Test details of 11m span beam

A 450mm deep asymmetric beam of 11.2m span with a ratio of flange areas of 1.5 was tested as being typical of composite fabricated beams of this span. The section comprised a 10mm thick top flange and web and 15mm thick bottom flange, and both flanges were 180mm wide and the plates were in S355 steel. The slab was 2.8m wide (= span/4). The beam was subject to 8 point loads to simulate uniform loading that was progressively increased in 7 cycles of loading to determine the effects of slip in the serviceability and ultimate load ranges.

The slab was 150mm deep and comprised an 80mm deep trapezoidal deck profile, as shown in Figure 3 (b). Shear connectors (125mm height and 19mm diameter) were welded through the decking at a spacing of 300mm. The characteristic resistance of the shear connectors was obtained as 75 kN from push tests on this deck profile (for a cylinder strength of 49 N/mm^2), and this was down–rated to 68 kN for the measured concrete strength of 29 N/mm^2 in the beam test.

A total of 16 shear connectors was placed from the support to the critical cross-section at the load point at 7/16 of the span, which is equivalent to a degree of shear connection of 33% for the measured steel yield strength of 410 N/mm^2. According to Eurocode 4, the minimum degree of shear connection should be 65% for this beam asymmetry and so the low degree of shear connection in the test could potentially lead to high slips and additional deflections.

A novel system of construction was used to ensure that the loads from the concrete slab were supported by the steel beam to mimic un-propped construction –see Figure 9. This was achieved by bolting channel section out-riggers to the web of the beam so that the outer edge of the decking was supported. The outer props were used to stabilise the beam during the concreting operation but were removed immediately after concreting. The out-riggers were removed when the concrete had gained its full strength. After concreting, the stress in the bottom flange due to the self-weight of the slab and beam was measured at about 29% of the steel yield strength.

Figure 9 *Outrigger beams used to support the decking to mimic un-propped construction (props used to stabilise the beam but were removed immediately after concreting)*

5.2 Test results for 11m span composite beam

Three cycles of increasing load were applied up to 50% more than normal service loading and a further 4 cycles were applied to failure. The maximum displacement at the end of the test was 220mm (=span/50). The beam after un-loading from the failure load is shown in Figure 10 in which the residual deflection was about 120mm.

Figure 10: *Deflection of composite asymmetric beam after unloading from the failure load*

The full load-deflection curve is shown in Figure 11, which indicated that full plasticity had developed at failure. The applied loading at failure was 18.0 kN/m^2 plus 3.6 kN/m^2 for the self-weight of the slab and beam and spreader beams. This was equivalent to a bending moment of 948 kNm. The predicted failure moment was 965 kNm, based on measured material strengths, which is only 2% higher and within the normal margin of acceptance for composite beam tests.

The load-deflection curve for the fifth load cycle up to 12 kN/m^2 expressed as a uniformly distributed load over the slab area is shown in Figure 12. This shows linear behaviour up to a load of 10 kN/m^2, which is twice the normal service loading and about 55% of the failure load. A permanent deflection of about 7mm was measured after unloading from this cycle, which indicates that irreversible deformation of the shear connectors had occurred.

The load-slip behaviour of the shear connectors is shown in Figure 13. Up to service load levels, the slip in the shear connectors was less than 0.5mm, and it was observed that slip increased more rapidly at a load of about 7 kN/m^2. At a load of 12 kN/m^2, the end slip was about 3 mm. The limiting slip of 6 mm for 'ductile' shear connectors was passed at a load of 15 kN/m^2, and from this point, slip increased rapidly to a maximum of 19mm at the end of the test, which demonstrated high deformation capacity.

The comparison of measured and theoretical deflections is made for a typical imposed load of 5 kN/m^2, for which the measured deflection of the beam was 16mm (taking account of the small residual deflection due to bedding-in of the loading system after the first load cycles). The calculation of the effective inertia and end slip is given below using the following data:

α_e = E_s/E_c = 6 (modular ratio for short-term loads)
A_s = 8.75×10^3 mm^2
A_c = $70 \times 2800 = 196 \times 10^3$ mm^2
I_s = 263×10^6 mm^4
I_c = 80×10^6 mm^4 (for h_c = 70mm)
y_s = 237 mm (measured from top flange of steel beam)

The composite second moment of area with rigid shear connectors is I_{comp} = 1131 x 10^6 mm^4. The shear connector stiffness per unit length is k/s_{sc} = 70/300 = 0.23 kN/mm^2. The effective second moment of area of the composite beam with flexible shear connectors is obtained from equation (11) as:

$$I_{eff} = 263x10^6 + \frac{80x10^6}{6} + \frac{(237+35+80)^2\left(\frac{196x10^3}{6}\right)}{\left[1+\frac{196x10^3}{6x8750}+\left(\frac{\pi}{11200}\right)^2\frac{210}{0.23}\left(\frac{196x10^3}{6}\right)\right]} = 839x10^6\ mm^4$$

The effective inertia I_{eff} is 74% of the composite inertia, I_{comp}. For an imposed load of 5 kN/m^2, the applied bending moment is 220 kNm and the end slip is obtained from equation (12) as 0.53mm. It follows that the maximum force in the shear connectors is: 0.53 x 70 = 37 kN at this load, and so the shear connectors are still elastic. Using this stiffness, the shear connectors would reach their elastic limit at a load of 9.2 kN/m^2, which agrees well with the test, and is 51% of the load acting on the composite beam at failure.

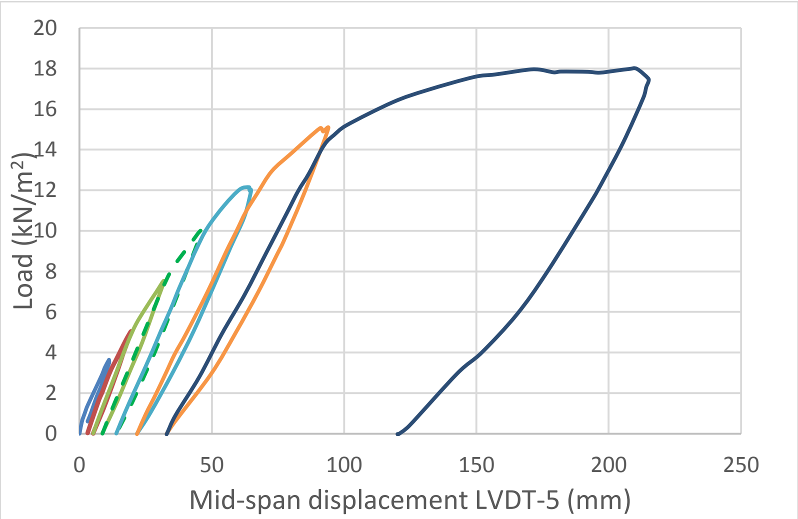


Figure 11 *Load-deflection curves of the asymmetric fabricated beam for 7 cycles of loading*

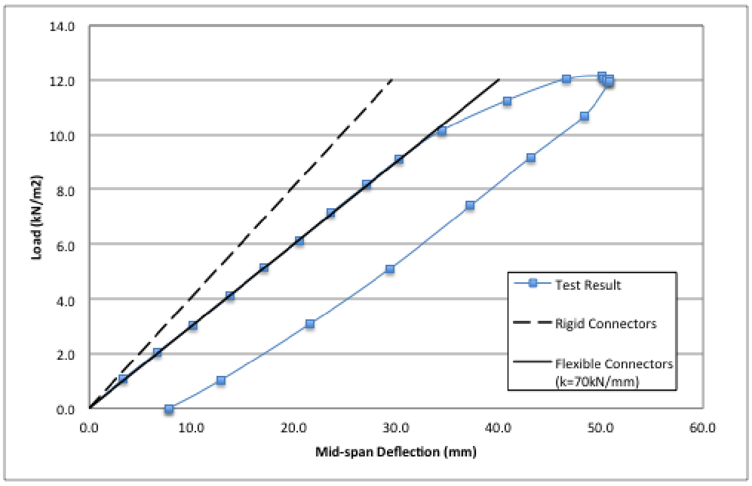


Figure 12 *Load-deflection cycle for the fabricated beam up to twice working load and comparison with the theory*

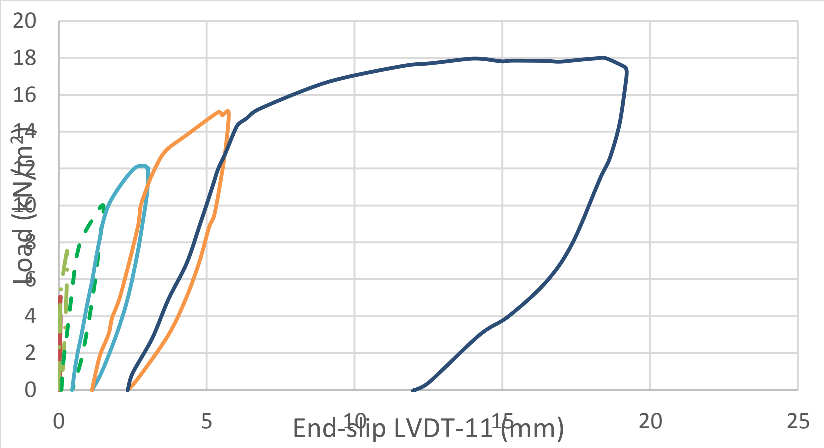


Figure 13 *Load-end slip curve for the side of the fabricated beam with maximum slip*

The calculated deflection of the fully composite beam with rigid shear connectors is 12mm and the deflection of the composite beam with flexible shear connectors (using *k*= 70 kN/mm) is 16.2 mm. Therefore, the additional deflection due to end slip is 35% of the deflection of the composite beam with rigid shear connectors. The test deflection was approximately 16 mm at this load, which is in good agreement with the theoretical deflection using this flexibility, as seen in Figure 12.

The test deflections at an equivalent imposed load of 5 kN/m^2 are compared in Table 3 with the theory presented in this paper and with the formulae in BS 5950-3 (BSI, 1990) and the American code (AISC, 2010). No formulae for additional deflections due to slip are given in Eurocode 4 and so the comparison is only with the deflection for the fully composite stiffness. The effective stiffness is calculated for two shear connector stiffnesses of k= 70 and 100 kN/mm, and the measured deflection is close to the stiffness corresponding to k= 70 kN/mm.

Table 3 *Comparison of the test results on 11m span beam with Code methods*

Test beam	Degree of shear connection in test	Measured deflection in test at 5 kN/m^2	Deflection of composite beam without slip	Eqn (17) using shear connector stiffness of:		Existing Code Methods	
				k = 70 kN/mm	k = 100 kN/mm	BS 5950-3	AISC Code
11.2m span asymmetric beams	33%	16.0mm	12.1mm	16.2mm	15.3mm	20.0mm	18.0mm

5.5 Comparison with short span beam tests

A series of 5 and 6m span beam tests was carried out at the Universities of Stuttgart and Luxembourg in 2014 as part of the same European research project, DISCCo -see acknowledgement. The tests on 5m span beams used IPE 300 sections and a 58 mm deep deck profile (see Figure 3(a)) with either single or pairs of 19 mm diameter shear connectors per deck rib. The tests on 6m span beams used IPE 360 sections and an 80 mm deep deck profile (see Figure 3(b)) with the same shear connector configurations. Only the 4 tests using 19mm diameter shear connectors welded through the decking are presented here. Other tests were carried out on shear connectors welded through holes cut in the decking and for 22mm diameter shear connectors, which are not reported here.

The tests were subject to two point loads placed at 37% of the span from the supports. The 6m span test is illustrated in Figure 14. The degree of shear connection (from the support to the point load) varied from 18 to 46% in the four tests based on measured yield strengths of 405 N/mm^2 and 382 N/mm^2 in the IPE 300 and IPE 360 beams respectively and for concrete cylinder strengths of 44 to 48 N/mm^2.

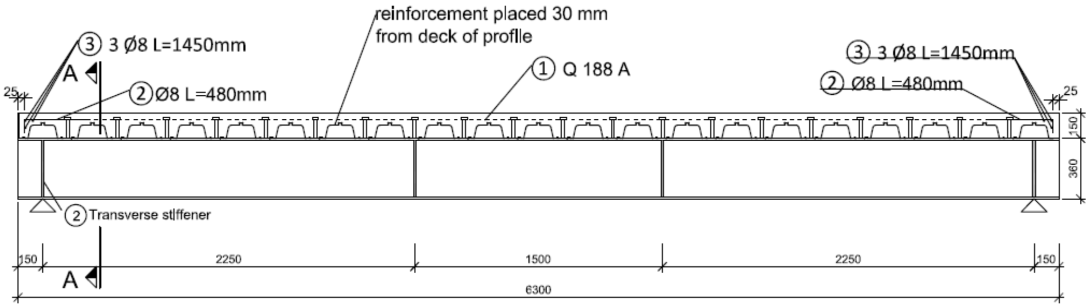


Figure 14: *Test on 6m span composite IPE 360 beam with single shear connectors per 80mm deep deck rib (load points at 2.25m from the supports)*

A typical graph of the 6m span beam in a displacement controlled test is shown in Figure 15. A representative point load of 100 kN (total load of 200 kN) is taken as being in the elastic range and this was about 45% of the eventual failure load.

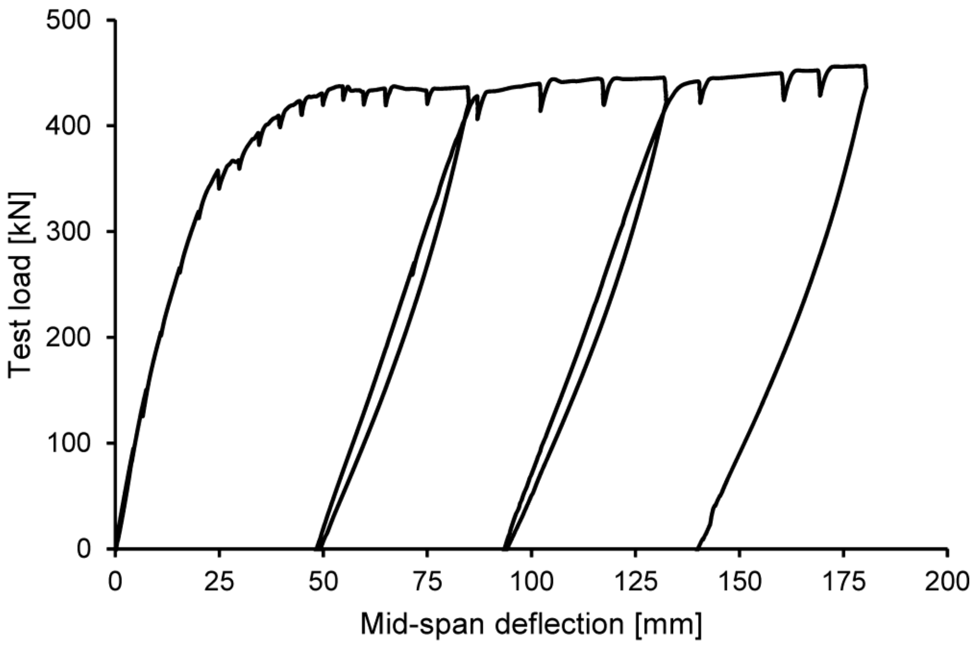


Figure 15: *Load-deflection curve for tests on 6m span composite IPE 360 beam with single shear connectors per 80mm deep deck rib*

The effective stiffness of the composite beams was calculated using k = 70 or 100 kN/mm for single and pairs of shear connectors respectively and the ratio of the effective stiffness to the fully composite stiffness was in the range of 61 to 73%. Deflections were calculated at mid-span for the 2 point loads case using the effective inertia calculated using equation (11).

Although the theory of effective stiffness was established for uniform loading, it is considered that the effective stiffness is also representative of the 2-point load case.

In this study of the serviceability performance of composite beams, two intermediate span beams were also analysed using the elastic theory in this paper. Two tests were carried out at Cambridge University in 1999 on 9m span composite beams subject to two point loads. The first used decking parallel to the beam and the second decking perpendicular to the beam. The beams were based on a modified 457 x191 x60 kg/m UB and the section was rolled as slightly asymmetric having a ratio of flange areas of 1.1 (but is treated as essentially symmetric). The deck profile was 60mm deep and the theoretical mid-span deflection was calculated for a shear connector stiffness of 70 kN/mm for transverse decking and 100 kN/m for parallel decking cases. Again, a point load of 100 kN was used in these comparisons which was about 45% of the failure load.

Table 4 presents the comparison between measured and theoretical deflections for all 6 tests, which shows that the comparison with the elastic theory using a representative shear connector stiffness is very good. End slips at this load level were measured as being small (approximately 0.3 mm), although the theoretical slip in the tests was calculated as being from 0.6 to 0.8 mm.

It is shown that the Code methods lead to slightly higher deflections for the low degrees of shear connection in the tests. The BS 5950-3 method over-predicts the test deflection of the beams by only 3 to 10% and is accurate for the shorter span cases. The AISC Code formula also has a similar range of accuracy.

Table 4 *Comparison of the serviceability test results on 5, 6 and 9m span beams with the theory and Code methods*

Beam Test Data	Shear Connectors	Degree of SC, η	Effective stiffness, I_{eff} (eqn (11))	Ratio I_{eff}/I_{comp}	Deflection at P = 100 kN		Comparison with Code methods	
					Test	Theory (eqn(11))	BS 5950-3	AISC
IPE 360 section, 6m span, 150 mm slab depth, 80 mm deck profile	Single at 300 mm	18%	366×10^3 mm^4	0.61	10.5mm	10.7mm	10.8mm	11.2mm
	Pairs at 300 mm	21%	405×10^3 mm^4	0.67	9.2mm	9.6mm	10.7mm	10.7mm
IPE 300 section, 5m span, 130 mm slab depth, 58 mm deck profile	Single at 207 mm	23%	210×10^3 mm^4	0.64	10.2mm	10.5mm	11.0mm	11.7mm
	Pairs at 207 mm	46%	240×10^3 mm^4	0.73	9.3mm	9.2mm	9.7mm	8.8mm
Asymmetric section based on 457x191UB, 9m span, 130 mm slab depth, 60mm deck profile	Single at 300 mm	37%	725×10^3 mm^4	0.71	17.2mm	17.0mm	17.7mm	16.8mm
	At 350mm - parallel decking	32%	736×10^3 mm^4	0.74	16.5mm	16.7mm	18.2mm	17.9mm

P = Point load of 100 kN (two point loads applied to beams) corresponding to approximately 0.4 P_u
η = Degree of shear connection in test (all tests with 19mm diameter shear connectors)
k = 70 kN/mm for single shear connectors
k = 100 kN/mm for pairs per deck rib and for parallel decking
I_{eff} = Effective second moment of area for flexible shear connectors of stiffness, k
I_{como} = Second moment of area of composite section for rigid shear connectors

6. Sensitivity Study on Deflections of Composite Beams of 6 to 18m Span

6.1 Additional Deflection due to Partial Shear Connection

To determine the effects of partial shear connection on deflections, a parametric study was carried out for beams with spans of 6 to 15m using typical section sizes and common design parameters. The beam sizes and typical spans for two span: depth ratios are presented in Table 5.

Table 5: *Beam sizes investigated in the sensitivity study on deflections due to partial shear connection*

Target Span	Beam Size	Common data
		Slab width = Span/4
6 to 7m	IPE 270	Slab depth = 130 mm
9 to 10m	IPE 400	Profile height = 60 mm
11 to 12.5m	IPE 500	Steel grade = S355
13.5 to 15m	IPE 600	Shear connectors at 300 mm spacing
17 to 19m	IPE750-137	Shear resistance, P_d = 70 kN
		Modular ratio, E_s/E_c = 10

The increase in deflection using the effective inertia in equation (11) is presented in Table 6 for these beams with two span: depth ratios and two degrees of shear connection. Single shear connectors at 300mm spacing with an elastic stiffness of 70 kN/mm and a shear resistance of 70 kN are consistent with 39% shear connection at the ultimate limit state to EN 1994-1-1. A pairs of shear connectors with a stiffness of k = 100 kN/mm per deck rib and combined shear resistance of 100 kN leads to 56% shear connection. Also presented in Table 6 in brackets are the deflections obtained from equation (1), which is given in the former BS 5950-3.

Table 6 *Increase in deflection of symmetric sections due to partial shear connection relative to that with rigid shear connectors using equation (11)*

Beam	Beam Span: Depth L/h = 22			Beam Span: Depth L/h = 25		
	I_{comp}/I_s	Shear connector spacing		I_{comp}/I_s	Shear connector spacing	
		Pairs at 300mm	Singles at 300mm		Pairs at 300mm	Singles at 300mm
IPE270	4.0	36% (40%)	48% (55%)	4.1	30% (41%)	41% (57%)
IPE400	3.2	26% (29%)	35% (40%)	3.3	22% (31%)	30% (42%)
IPE500	2.9	21% (25%)	29% (34%)	3.0	18% (27%)	25% (36%)
IPE600	2.7	19% (23%)	25% (31%)	2.8	16% (24%)	22% (33%)
IPE750	2.6	14% (21%)	19% (29%)	2.7	11% (23%)	16% (30%)

Increase in deflection expressed as a % relative to composite beam with rigid shear connectors
Figures in brackets are from equation (1) to the former BS 5950-3

It is apparent that the additional deflection due to the flexibility of the shear connectors reduces with beam span because the proportionate stiffness of the composite section to the steel section reduces with span. The proportionate increase in deflection due to end slip also decreases with the beam span: depth ratio as shear becomes more significant. The comparison with equation (1) to BS 5950-3 is also given in this table based on the degree of shear connection. It can be seen that the comparison with the elastic theory is reasonably close for shorter span beams with a span; depth ratio of 22, but equation (1) becomes increasingly conservative for longer span beams with higher span: depth ratios.

6.2 Finite element analysis of composite beams

The generalised finite element package *ANSYS*, was used to model the composite beams. The FE model was built using shell elements for the steel section and solid elements for the slab. The shear connectors were modelled using nonlinear spring elements. The FE model mesh is shown in Figure 16. The deck ribs were modelled by a reduced stiffness (elastic modulus of $E_c/10$) compared to the rest of the slab. This was done to avoid overestimating the system stiffness, since the contribution from the concrete ribs is already accounted for in the load-slip characteristics which are input to the spring stiffness only. Half the beam span was modelled because of symmetry and the ends of the beams were taken as simply supported.

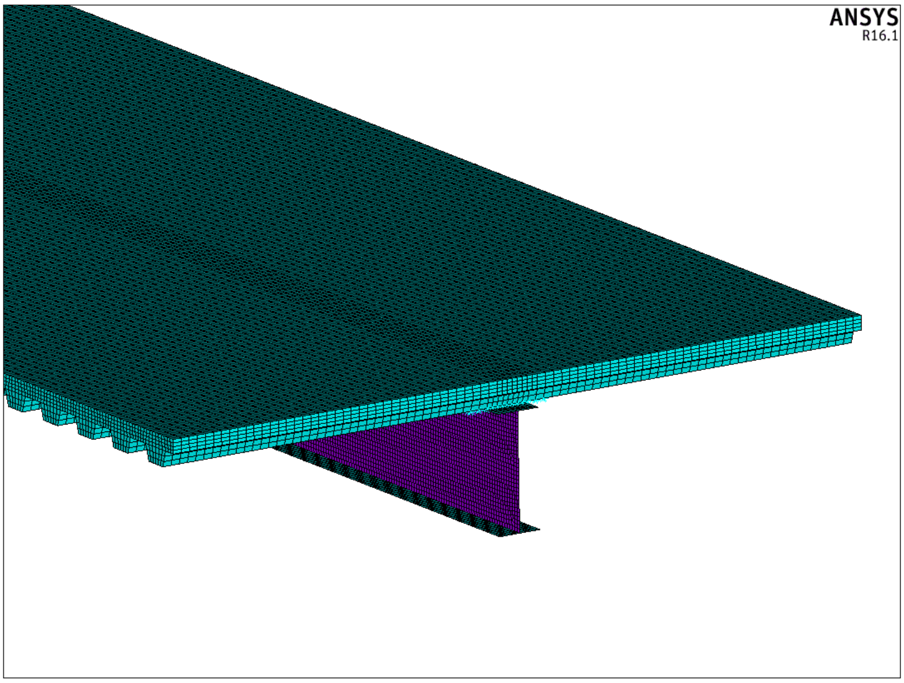


Figure 16: *FE model of the composite beam showing the mesh detail*

The FE model was first verified against the long span composite beam test. The material model assumed for the steel beam was bilinear (linear up to the measured yield strength and then a plateau with strain hardening gradient of E/100). A trilinear model was used for modelling the concrete which does not capture the unloading behaviour occuring at strains higher than 0.2% but this concrete strain was not exceeded due to the low degrees of shear connection. The shear connectors (springs) were modelled in a bilinear form with an elastic stiffness of 70kN/mm and a plastic plateau at a shear connector resistance of 70 kN based on push-out test results.

The load versus deflection graph obtained from the model in comparison to the beam test results is shown in Figure 17, which is in close agreement in the initial load range and close to failure. The load versus slip graph is shown in Figure 18, which shows that the end slip is lower than the FE prediction in the initial load range. This demonstrated the adequacy of the FE model, which was the used to carry out a parametric study of the serviceability performance of a range of beams of 6 to 18m.

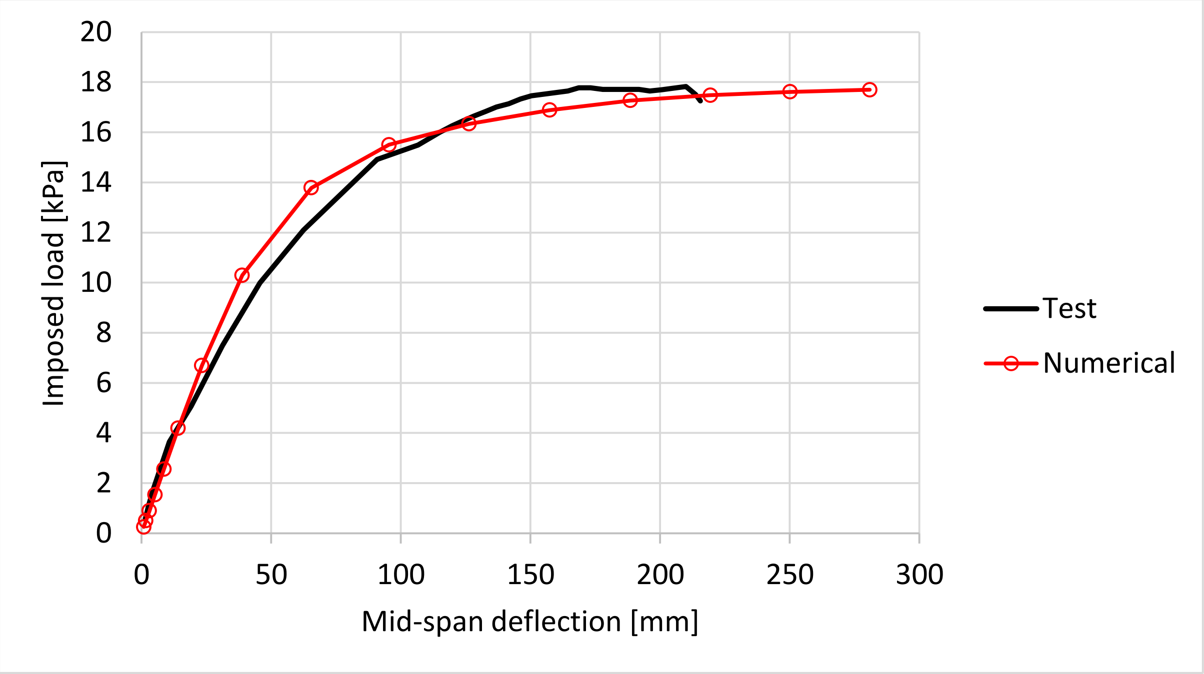


Figure 17: *Load versus deflection for the 11.2 m span asymmetric fabricated beam obtained from the test and FE simulations*

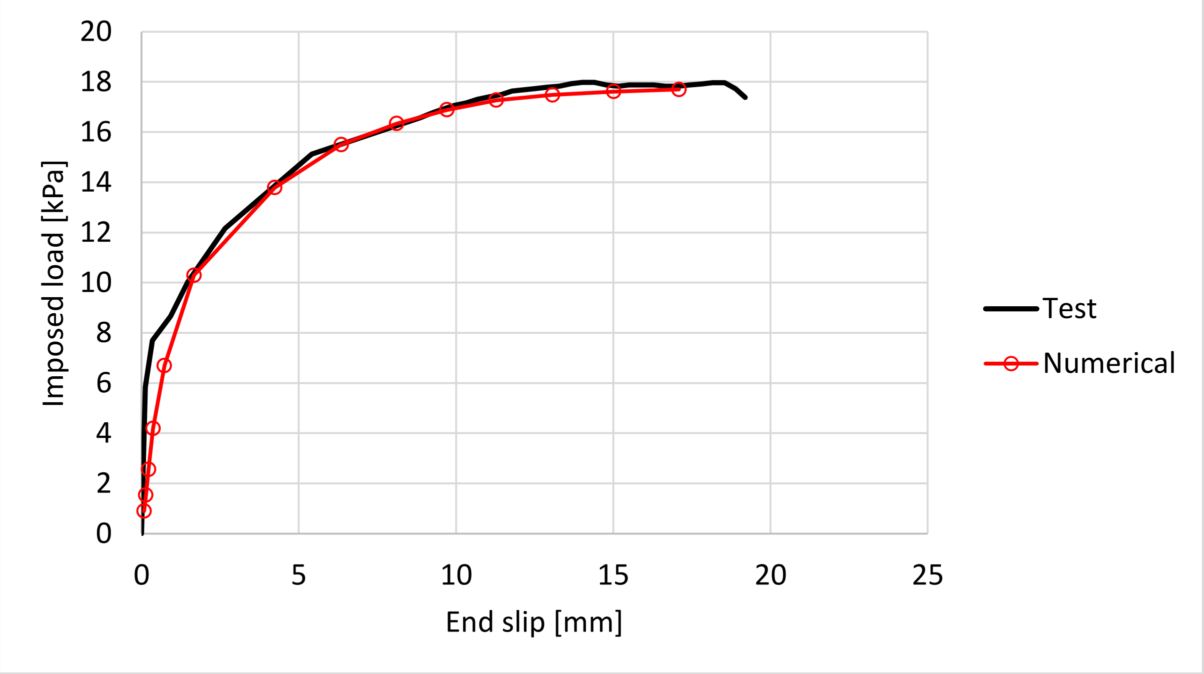


Figure 18: *Load versus end slip for the 11.2 m span asymmetric fabricated beam obtained from the test and FE simulations*

Beams using rolled IPE 270 to IPE 600 sections with span of 6 to 15m were modelled. An imposed load of 0.3 P_u was taken as being typical of the service load for un-propped composite beams, and a load of 0.5 P_u for propped composite beams. It is shown in Table 7 that for un-propped beams, the end slip varies only slightly with beam span and does not exceed 0.7mm at this load, which is within the elastic range. However, for propped beams, the end slip at a load of 0.5 P_u increased from 1.1 to 1.5 mm for 9 to 15m spans, which is the post–elastic range and would lead to higher deflections due to partial shear connection.

The FE results are also compared to the elastic theory in this paper for the two load levels. It is shown that the comparisons for un-propped beams and for shorter span propped beams are within 10% and the difference is likely to be more in the FE modelling of the stiffness of the shear connection system than in the theory. The FE analysis leads to higher deflections for longer span propped beams due to the plasticity developed in the shear connectors.

Table 7: *Deflection of composite beams at 30% and 50% of their maximum load, P_u based on finite element results compared to elastic theory*

Span [m]	Section	P_u [kN/m]	Method	FE result and elastic theory for deflection and end slip			
				Load of 0.3P_u		Load of 0.5P_u	
				δ [mm]	s [mm]	δ [mm]	s [mm]
6	IPE270	68.6	FE prediction	11.4	0.53	19.0	0.88
			Theory (eqn (11))	10.6	0.55	18.2	0.91
9	IPE400	70.4	FE prediction	17.4	0.60	29.5	1.12
			Theory (eqn (11))	16.0	0.61	26.6	1.02
12	IPE500	64.9	FE prediction	24.5	0.63	41.7	1.28
			Theory (eqn (11))	22.0	0.64	36.7	1.06
15	IPE600	64.1	FE prediction	32.3	0.66	55.5	1.48
			Theory (eqn (11))	29.0	0.67	48.4	1.12

P_u is the maximum load acting on the beam determined from its plastic bending resistance
δ is the deflection of the composite beam at this load and s is the end slip

6.2 Sensitivity study on minimum degree of shear connection

The finite element analyses were extended to consider at what point a limiting slip of 1mm is exceeded for beams of 6 m to 15 m span with a span: depth ratio in the range of 22 to 25. The working loads for these beams were taken as a proportion of their plastic bending resistance for the particular degree of shear connection. The analyses are repeated by adjusting the degree of shear connection so that the end slip did not exceed 1 mm.

The serviceability load in this analysis was first determined from: (M_{pl} –1.35M_{sw})/1.5 for un-propped beams and M_{pl}/1.5 for propped beams, where M_{pl} is the plastic resistance of the beam for partial shear connection and M_{sw} is the moment due to the self-weight of the beam and slab. This leads to serviceability loads that are higher than 0.3P_u and 0.5P_u considered earlier. The results in terms of the minimum degree of shear connection for 1mm end slip are presented in Table 8 for symmetric beams.

It is argued that the statistical likelihood of repeated loading of the magnitude of the full serviceability load is low and therefore the cumulative effects of slip will not be significant. Therefore the load at which the slip is calculated is taken as: 0.8 x (M_{pl} –1.35M_{sw})/1.5 for un-propped beams and 0.8M_{pl}/1.5 for propped beams. This leads to serviceability loads closer to 0.3 P_u and 0.5P_u for un-propped and propped beams respectively. For symmetric beams, the minimum degree of shear connection for a 1mm slip limit at the serviceability limit state was found to be in the range of 0.29 to 0.47 for propped beams and 0.23 to 0.32 for un-propped beams (i.e. about 20% less than for propped beams).

Based on these results for a maximum serviceability slip of 1mm, it is proposed that the cut-off in the minimum degree of shear connection for symmetric composite beams could be taken as 0.3 for un-propped beams and 0.4 for propped beams. The 0.4 minimum limit is the same as for the existing shear connection rules in Eurocode 4, and so a relaxation in the minimum degree of shear connection is made only for un-propped beams.

Table 8 *Minimum degree of shear connection for symmetric beams based on 1mm slip limit at the serviceability limit state*

Span [m]	Beam (in S355 steel)	Number of shear connectors per deck rib	Un-propped beams		Propped beams	
			Minimum degree of shear connection for 1mm slip calculated for a serviceability moment of:			
			0.8(M_{pl} – 1.35M_{sw})/1.5	(M_{pl} 1.35M_{sw})/1.5	0.8M_{pl}/1.5	M_{pl}/1.5
6	IPE270	1	0.26	0.37	0.29	0.40
		2	0.32	0.43	0.37	0.48
9	IPE400	1	0.27	0.37	0.34	0.46
		2	0.32	0.46	0.39	0.53
12	IPE500	1	0.26	0.37	0.37	0.48
		2	0.32	0.43	0.43	0.56
15	IPE600	1	0.23	0.35	0.38	0.50
		2	0.30	0.43	0.47	0.55

Shear connector stiffness of 70 kN/mm for single shear connectors and 100 kN/mm for pairs of shear connectors
M_{pl} is the plastic resistance of the beam for partial shear connection and M_{sw} is the moment due to the self-weight of the beam and slab

The same approach was used for asymmetric composite beams with a maximum ratio of flange areas of 3:1 in which the thickness of the original top flange was multiplied by 0.5 and the thickness of the original bottom flange was multiplied by 1.5 in order to keep to the same cross-sectional area as the original beam.

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In this case the minimum degree of shear connection is higher than for symmetric sections. Results of these analyses are presented in Table 9. For propped beams, the minimum degree of shear connection is in the range of 0.40 to 0.58 and for un-propped beams is in the range of 0.37 to 0.46. It follows that the minimum degree of shear connection of an un-propped beam is about 10% less than for a propped beam.

As a good approximation for un-propped beams, the minimum limit on the degree of shear connection to control slip at the serviceability limit state may be taken as:

$$\text{Min } \eta = 0.2 + 0.1(A_{fb}/A_{ft}) \geq 0.3 \qquad (15)$$

where A_{fb}/A_{ft} is the ratio of flange areas

Similarly for propped beams, the minimum limit on the degree of shear connection to control slip at the serviceability limit state may be taken as:

$$\text{Min } \eta = 0.3 + 0.1(A_{fb}/A_{ft})) \geq 0.4 \qquad (16)$$

These cut-off values should be applied to the general rules for the minimum degree of shear connection at the ultimate limit state irrespective of the limiting end slip of 6 or 10mm.

Table 9 *Minimum degree of shear connection for asymmetric beams based on original IPE section with flange areas of 3:1 based on 1mm slip limit at the serviceability limit state*

Span [m]	Asymmetric beam based on original section (in S355 steel)	Number of shear connectors per deck rib	Un-propped beams		Propped beams	
			Minimum degree of shear connection for 1mm slip calculated for a serviceability moment of:			
			$0.8(M_{pl} – 1.35M_{sw})/1.5$	$(M_{pl}\ 1.35M_{sw})/1.5$	$0.8M_{pl}/1.5$	$M_{pl}/1.5$
6	IPE270 with A_{fb}/A_{ft} =3	1	0.37	0.48	0.40	0.51
		2	0.43	0.53	0.48	0.59
9	IPE400 with A_{fb}/A_{ft} =3	1	0.39	0.54	0.46	0.61
		2	0.46	0.60	0.53	0.68
12	IPE500 with A_{fb}/A_{ft} =3	1	0.38	0.51	0.48	0.60
		2	0.43	0.59	0.53	0.69
15	IPE600 with A_{fb}/A_{ft} =3	1	0.38	0.51	0.50	0.64
		2	0.43	0.58	0.58	0.73

Data the same in Table 8

Conclusions

It is shown from a series of 7 short and long span beam tests that the effective second moment of area of a composite beam with flexible shear connectors may be calculated using equation(11) as a function of the shear connector stiffness, *k*, and longitudinal spacing, s_{sc}:

For un-propped beams, it is shown by comparison with long span beam tests that deflections may be calculated with reasonable accuracy using *k*= 70 kN/mm for single shear connectors per deck rib and *k*= 100 kN/mm for pairs of shear connectors per deck rib. The end slip may be presented as a function of the inertias of the composite section, I_{comp}, and the steel section, I_s, as in equation (12).

The comparison of the additional deflections due to partial shear connection between the tests and the former BS 5950-3 and the AISC Code were found to be reasonably accurate for the shorter span beams, but conservative by about 20% for the 11m span asymmetric beam test with 33% shear connection. Although this is work in progress, it is proposed that the former BS 5950-3 method may be modified to improve its accuracy for long span beams by taking into account the span: depth ratio of the beam. Therefore the deflection of un-propped composite beams may be determined with reasonable accuracy from:

$$w = w_c + 0.3\,(20/(L/h))^{0.5}\,(1-\eta)\,(w_s - w_c)$$

where *L* is the beam span and *h* is the steel beam height and η is the degree of shear connection at the ultimate limit state and w_s and w_c are the deflections using beam stiffness, I_s and I_{comp} respectively at the serviceability loads

A minimum cut-off in the degree of shear connection is required to control the end slip at the serviceability limit state so that cumulative deflections do not occur under repeated loading. For un-propped beams, it is proposed that this minimum limit is taken as:

$$\eta = 0.2 + 0.1(A_{fb}/A_{ft}) \geq 0.3$$

where A_{fb}/A_{ft} is the ratio of flange areas

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